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A Study on the Dynamic Dimensionless Behaviours of Underground Pipes Due to Blast Loads Using Finite Element Method

Akinola Johnson Olarewaju

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1. Introduction

In our day to day activities in engineering field, underground structures such as pipes, shafts, tunnels, tanks, etc. are encountered. These are used for services such as industrial services, underground domestic services, mining and agricultural engineering services, onshore and off-shore engineering projects, sewerage, domestic and industrial water supplies, liquid gas, acid, gas in petro-chemical industries, industrial and domestic wastes, to mention a few [21]. Elements of target during wars and civil unrests are industrial centres, military installations, oil and gas pipelines, centres of communication, defence control centres, to mention a few depending on the functions and importance of the facility. Blast characteristics as one of the constituents of blast have to do with explosion [11].

Apart from wars, civil unrests, terrorist attacks and other accidental explosions, in the oil and gas industry, the main sources of blast includes the accumulation of explosive gas in pipes during two phase flow in liquid and gas especially in bends as well as leakage from reciprocating compressor catching fire and exploding. It has been reported that blast can create sufficient tremors to damage substructures over a wide area. Blast at Fukushima nuke plant in Japan was felt 40 km away from the source. Blast could be thought of as an artificial earthquake with a short duration (i. e. transient), that is, blast is a short discontinuous event. In the manufacturing industries, it leads to disruption in production, land degradation, air pollution, and so on.

Due to the immeasurable consequences of blast mentioned above as a result of earth tremor there is need to mitigate the consequences of blasts in underground structures such as pipes. Mitigation measure could be in the form of designing underground structures to resist the effects of blast or by repairing those damaged by blast. The categories of blast that are



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applicable to underground structures like pipes are; (1) underground blast, (2) blast in open trench, (3) internal explosion inside the pipes as well as (4) surface blast [11, 27]. These are graphically shown in Figure 1 a to d [13, 10].

In view of this, ground shock parameters which are equally known as the soil movement parameters translate into loading which the soil delivers to the buried structures. These among other parameters are required to be estimated for saturated clay and sand/soils in surface blast and underground blast using [32] and empirical methods respectively. For surface blast, the parameters are: (i) peak reflected pressures, (ii) peak overpressure, (iii) specific impulse, (iv) shock front velocity, (v) horizontal and vertical displacement, (vi) horizontal and vertical velocity, (vii) horizontal and vertical acceleration and (viii) arrival time. For underground blast, the parameters are: (i) peak particle displacement, (ii) peak particle velocity, (iii) loading wave velocity, (iv) specific impulse, (v) side-on overpressure, (vi) peak reflected pressure, etc. Details of these could be found in [10, 11, 7, 18] and [20]. The loading wave velocity is a function of both the peak particle velocity and the seismic velocity. It is high at short range due to high values of particle velocity but as the range increases it reduces to the seismic velocity of soil [5, 13]. This study is aimed at (1) the determination of blast load parameters for various blast scenarios considered, (2) the determination of the magnitude of the dynamic and dimensionless responses between the various components of blast as well as carrying out parametric studies of the response of underground pipes due to blast loads and (3) the establishment of design parameters for the design of underground pipes.



Figure 1. Categories of blast applicable to underground pipes [10]

2. Background study

Little work has been done on the behaviours of underground structures like pipes due to blast loads with a few to providing design parameters and guidelines for the design of underground pipes to resist effects of blast loads. This is important most especially when different categories of blast (mentioned earlier) applicable to underground structures are to be considered. For each category of blast, various blast load parameters are required. These depend on the categories of blast that are applicable to underground structures (in this case, pipes). Blast related problems are impact problem that could be solved and studied using finite element based numerical code [1, 3].

Underground structures generally are made of steel, cast-iron, ductile steel, reinforced concrete, polyvinylchloride, clay, glass fibre, etc. manufactured to different standard sizes and thicknesses according to the available codes of practice. The major difference in structural materials as well as ground media is the difference in the values of their Young's modulus, Poisson's ratio and density. High stiffness structural materials have higher values of Young's modulus, e. g. steel structures, concrete structures, etc. while low stiffness structural materials have low values of Young's modulus, e. g. polyethylene structures, vitrified clay structures, etc. [8]. There are various types of structural materials that are commonly used, these among others are: 1) Reinforced Concrete Structures - these are made of aggregate (coarse and fine), cement, water and reinforcement. The average density is 2500 kg/m³ and Young's modulus is 28 x 10³ N/mm². The average Poisson's ratio is 0.175. 2) Cast Iron or Grey Iron Structures - these types of structures are strong but brittle, though it offers a long service life and equally maintenance free. The density ranges from 7800 kg/m³ to 7950 kg/m³, Young's modulus, E is 200 x 10³ N/mm³ (MPa) and Poisson's ratio is 0.31. 3) Ductile Iron - these types resemble cast iron structures in characteristics, but ductile iron structures are stronger and tougher than cast iron materials. The density ranges from 7800 kg/m³ to 7950 kg/m³, Young's modulus, E is 200 x 10³ N/mm² (MPa) and Poisson's ratio is 0.31. 4) Steel - these are manufactured using steel plates. The density is 7950 kg/m³, Poisson's ratio ranges from 0.2 to 0.3 while the Young's modulus is 200×10^3 N/mm². 5) Polyethylene - these are made of resin materials. The density is 9.55 x 10⁻⁴ kg/m³ and modulus ranges from 800 N/mm² to 1100 N/mm². 6) Vitrified Clay (VC) - sewerage clay structures are produced from an inert material, clay, fired to vitrification state to last for a long time. The density is 2420 kg/m³, average Young's modulus is 35 GPa and Poisson's ratio ranges from 0.15 to 0.20 [10].

In the behaviour studies of underground empty pipes due to blast loads, various parameters are involved in the analysis. These parameters are Young's modulus of the ground medium (i. e. loose sand, dense and undrained clay as the case may be), Poisson's ratio of the ground medium, density of the ground medium, unit weight of the ground medium, Young's modulus of structures, Poisson's ratio of structures, density of structures, unit weight of structures, thickness of structures, diameter of structures, depth of burial of structures, length of structures, size (length, breadth and depth) of the ground medium, contacts between the ground media and structural materials, volume change in structures, blast loads (pressure, loading wave velocity, specific impulse, etc.), observed parameters (displacement, pressure, stress and

strain as the case may be), round off, truncation, large number of equations and parameters, ill conditioned matrices etc. in the analysis [6]. Due to the involvement of these parameters in the analysis, there is bound to be variations in the solutions and results obtained. One of the powerful modelling tools used to infer some information about the variations in the solutions and results is dimensional analysis. In dimensional analysis, a constant is characterized by

where L (length), M (mass), and T (time) is the dimension of a physical quantity which would have a dimensionless quantity, i.e. $[\phi] = 1$, K and a, b, c are constants. This property is called dimensional homogeneity which forms the key to dimensional analysis [4].

(1)

 $[\varphi] = KL^a M^b T^c$

The non-linearity of a problem are basically in the definition of material, contact problems, large displacements and rotations due to large loads (i. e. non-linear geometry) and time incrementation to ensure stability. Among other numerical methods, finite element is a powerful tool that could be used to model and analyze complex dynamic structure interaction among which is soil structure interaction during accidental explosion. The response at any desired point of the structure can easily be determined. Most soils are homogenous, isotropic and anisotropic. In this study, the soil and structures will be considered as linear elastic, homogeneous, isotropic material. For such material, only two elastic constants are needed to study the mechanics/behaviours of such body [5].

This study is aimed at determining the dimensionless behaviours of empty underground pipes due to blast loads by simulation using finite element method. This is with a few to providing parameters and guidelines for the design of underground pipes to resist the impact of blast loads [13]. Consequently, environmental risks and hazards caused by blast would be greatly reduced if not completely eliminated.

3. Methodology

3.1. Determination of ground movement parameters for surface blast, open trench blast and internal explosion

In this study, [32] was used to predict positive phase of blast loads at various stand-off points. Since pressure is the determining factor in the design and behaviour study of underground structures, the side-on overpressure and peak reflected pressure for explosives ranging from 10 kg TNT to 10000 kg TNT were determined. Details of these could be found in [27, 9, 10], and [11].

3.2. Determination of ground movement parameters for underground blast

In order to determine ground shock parameters, analytical methods were used. The parameters thus determined are peak particle displacement, peak particle velocity, loading wave

velocity (for sand and saturated clay), side-on overpressure (for sand and saturated clay) and specific impulse for sand and saturated clay [32]. Soil test results are required in the final design to accurately determine the density and loading wave velocity of the particular soil at the site. However, since the range of seismic velocities obtained from [32] is so large, the lower bound value of the velocity was used as recommended to produce a conservative estimate of the induced motion. Details of this could be found in [26, 27, 9] [10] and [11].

3.3. Analysis of the constituents of blast

Direct-integration dynamics of time integration in the explicit integration scheme of central difference of numerical method was used to solve the equations of motion (Equations 2) of the system. This is integrated through time.

$$[m][U] + [c'][U] + [k][U] = [P] \approx [m] \frac{d^2x}{dt^2} + [c'] \frac{dx}{dt} + [k][x] = F(t) \approx [m]f'' + [c']f' + [k]f = F(t)$$
(2)

for
$$U_{(t=0)} = U_0$$
 and $U_{(t=0)} = U = V_0$ (3)

$$P = m \frac{x_{(1+1)} - 2x_i + x_{(i-1)}}{(\Delta t)^2} + c \frac{x_{(1+1)} - x_{(1-1)}}{2\Delta t} kx_i = Pi$$
(4)

where Ui = U(t) and U_{+1} can be written as

$$U_{(i+1)} = \frac{1}{\frac{m}{h^2} + \frac{c}{2h}} \left[\left\{ \frac{2m}{h^2} - k \right\} U_i + \left\{ \frac{c}{2h} - \frac{m}{h^2} \right\} U_{(i-1)} + P_i \right]$$
(5)

where m = mass of element, c = element damping, k = stiffness matrices, t = time, U = displacement, P = load vectors and dot = time derivatives. \ddot{U}_o is known from the given initial conditions while i is the increment number of an explicit dynamic step [5]. The terms $_{i+1}$ and $_{i-1}$ refers to midincrement values. The time duration for the numerical solution could be divided into intervals of time Δ t (h). It should be noted that with no damping

$$\Delta t \le \frac{2}{\omega \max} \tag{6}$$

for stable and satisfactory solution or with damping

$$\Delta t \le \frac{2}{\omega \max} \left(\sqrt{1 + \xi \max^2 - \xi \max} \right) \tag{7}$$

$$\ddot{U}o = (m)^{-1}(Po - c\dot{U}o - kUo)$$
 (8)

$$U_{-1} = Uo - h\ddot{U}o + \frac{h^2}{2}\ddot{U}o$$
(9)

 ω max is the maximum natural frequency, ξ max is the critical damping factor. Stability limit is the largest time increment that can be taken without the method generating large rapid growing errors. The accuracy of the solution depends on the time step Δ t = h. However, there are some conditionally stable methods where any time step can be chosen on consideration of accuracy only and need not consider stability aspect [2, 5]. Using the explicit integration scheme to solve the above equation of motion makes it unnecessary for the formation and inversion of the global mass and stiffness matrices [M, K]. It also simplifies the treatment of contact between the constituents of blast and requires no iteration. This means that each increment is relatively inexpensive compared to the increments in an implicit integration scheme. It also performs a large number of small increments efficiently.

Explicit integration scheme are used for the analysis of large models with relative short dynamic response times and extremely discontinuous events or processes. This makes it relevant and justifiable to be used for the analysis of the study of the response of underground structures due to blast loads because blast is a short discontinuous event [1, 2, 3]. For convenience, dynamic equations are written as

$$M U = P - I \tag{10}$$

These complete general equations apply to the behaviour of any mechanical system that contains all nonlinearities such as large deformations, nonlinear material responses and contact problems. When the inertial or dynamic force is small enough, the equations reduces to the static form of equilibrium [1]. In solving the equation of motion in relation to dynamic soil-structure problems such as accidental explosion, explicit dynamics is a mathematical technique for integrating the equations of motion throughout time. The explicit dynamic integration method is also known as the forward Euler or central difference algorithm where unknown values are obtained from information already known [1].

By adding or combining the explicit dynamic integration rule with elements that use a lumped mass matrix is what makes an explicit finite element program work. The lumped mass matrix, *M*, allows the program to calculate the nodal accelerations easily at any given time, *t*, using the following expression:

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$$\ddot{U} = M^{-1}(P - I)$$
 (11)

where *P* is the external load vector, *I* is the internal load vector, *u* is the displacement and dot indicate time derivative [1, 2]. ABAQUS package was used to solve the equations of motion (Equation 2) of the system with the initial conditions. The time duration (period) [13] for the numerical solution [2, 5] was divided into intervals of time Δ t (h), where h is the time increment.

In ABAQUS/Explicit which is used in this study, the time incrementation is controlled by the stability limit of the central difference operator. The time incrementation scheme is fully automatic and requires no user intervention. User-specified time incrementation is not available because it would always be non-optimal. Standard and Explicit integration schemes in ABAQUS CAE are two separate program modules having data structures different from each other. Therefore, the explicit dynamics procedure cannot be substituted or used in the same analysis as any of the procedures in Standard program of ABAQUS CAE [1, 2]. The explicit dynamics analysis procedure in [1] is based upon the implementation of an explicit integration rule together with the use of diagonal element mass matrices. The equation of motion referred to in Equation 2 for the body is integrated throughout time using the explicit central difference integration rule:

$$\dot{U}^{(i+0.5)} = \dot{U}^{(i-0.5)} + \frac{\Delta t^{(i+1)} + \Delta t^{(i)}}{2} \ddot{U}$$
(12)

$$U^{(i+1)} = U^{(i)} + \Delta t^{(i+1)} \dot{U}^{(i+0.5)}$$
(13)

where U = velocity, U = acceleration, superscript (i) = increment number and (i- $\frac{1}{2}$) and (i+ $\frac{1}{2}$) = mid-increment values [36]. The central difference integration operator is explicit because the kinematic state can be advanced using known values of U and U from the previous increment [2]. The integration rule of the explicit in ABAQUS CAE is quite simple but by itself does not provide the computational efficiency associated with the explicit dynamics procedure. The key to the computational efficiency of the explicit procedure is the use of diagonal element mass matrices because the inversion of the mass matrix which is used in the computation for the accelerations at the beginning of the increment is triaxial.

$$\ddot{U}^{(i)} = M^{-1}(F(i) - I^{(i)})$$
⁽¹⁴⁾

where M = diagonal lumped mass matrix, F = applied load vector, and I = internal force vector. The explicit procedure requires no iteration and no tangent stiffness matrix [1, 37]. For initial conditions, certain constraints, and presentation of results, special consideration and/or treatment of the mean velocities $U^{(i+0.5)}$, $U^{(i-0.5)}$, etc. is required. For presentation of results, the state velocities are stored as a linear interpolation of the mean velocities

 $\dot{U}^{(i+1)} = \dot{U}^{(i+0.5)} + 0.5\Delta t^{(i+1)} \ddot{U}^{(i+1)}$ (15)

By itself, the central difference operator is not self-starting. This is because the value of the mean velocity $U^{(-1)}$ needs to be defined. The initial values (that is, at time t=0) of velocity and acceleration are set to zero (0) unless they are specified or indicated by the user. Under this condition,

$$U^{(+0.5)} = U^{(0)} + \frac{\Delta t^{(1)}}{2} U^{(0)}$$
(16)

Substituting this expression into the update expression for $U^{(i+0.5)}$ yields the following definition of $U^{(-0.5)}$ [2],

$$\dot{U}^{(-0.5)} = \dot{U}^{(0)} - \frac{\Delta t^{(0)}}{2} \ddot{U}^{(0)}$$
(17)

Stability limit is the largest time increment that can be taken without the method generating large rapid growing errors. The accuracy of the solution depends on the time step $\Delta t = h$. However, there are some conditionally stable methods where any time step can be chosen on consideration of accuracy only and need not consider stability aspect [1, 2, 5, 15].

3.4. Internal explosion

In the study of the behaviours of underground pipes due to internal explosion, the range of explosives considered for 0.8 m, 1.0 m, and 1.2 m diameter pipes are 50 kg TNT, 100 kg TNT and 250 kg TNT. For all these pipes and explosives, for a conservative estimate, the design curve obtained from [32] was used to determine the blast wave parameters for different explosives in different sizes of pipes. The finite element model of all the infinite soils considered is 21.04 m width, 8.04 m depth and 20 m long. Steel and concrete pipes are 1.0 m diameter and 20 m long and thickness of 0.010 m and 0.020 m were considered. Pipes were laid horizontally each at 3.04 m and 6.04 m depths in other to consider various embedment. It is assumed that there is no slip between the pipes and soil and as a result, there is perfect bond between the pipes and the soil. The boundary condition of the model was defined with respect to global Cartesian axes in line with [3]. The pressure was taken to be the normally reflected pressure.

The explosion was assumed to explode at the centre right inside pipe with the centre coinciding with the centre of the pipe. The soil and pipe properties as revealed by several researchers and investigators were used [5, 35].

According to [29], it was concluded that the soils with montimorillonite (clay) as a dominant mineral are more susceptible to durability problems in particular when these soils are exposed to volume changes caused by swell and shrink related volume changes. This makes it necessary for the consideration of undrained clay in this study of the response of underground pipes due to blast loads. In Malaysia, soils encountered were slightly variable, with a mixture of loose sands, very stiff soils and very soft clays as deep as 24 m. Since Malaysia is in the tropical region with unpredicted rainfall, it is justifiable to consider undrained behaviour of clay. Details of this could be found in [23] and [25].

3.5. Underground blast and open trench blast

Using the estimated loading wave velocities from [32], behaviours of modelled underground steel and concrete pipes buried in loose sand, dense sand and undrained clay (Figure 2 a and b) at different embedment ratios were examined using ABAQUS/Explicit. Infinite ground media of 100 m long, 100 m width and 100 m depth were modelled (Figure 2b). Buried steel and concrete pipes of 100 m long, thicknesses of 10 mm and 20 mm each having 1 m external diameter buried at height/depth ratio of 1, 2, 3, 4 and 5 from the ground surface and surrounded by infinite intervening medium of 1 m internal diameter and 0.15 m thick were modelled. The schematic diagram of the behaviour of underground pipe due to underground blast and open trench blast are shown in Figure 1 a and b. Details of these could be found in [16] and [17].

3.6. Surface blast

Infinite ground media of 100 m long, 100 m width and 100 m depth were modelled. Underground steel and concrete pipes of 100 m long, thicknesses of 10 mm and 20 mm each having 1 m external diameter buried at height/depth ratio of 1, 2, 3, 4 and 5 from the surface of the ground and surrounded by intervening medium of 1 m internal diameter and 0.15 m thick were modelled (Figure 2 a and b). The diagram of underground pipe due to surface blast is shown in Figure 1d. Full analysis was carried out using ABAQUS/Explicit. Details of this could be found in [9, 18] and [24].

3.7. Parametric studies

The parametric studies on the behaviour of underground pipes due to blast loads considered are the effects of parameters such as Young's modulus of soil, Young's modulus of pipe, blast loads, pressure, deflection, depth, etc. at the crown, invert and spring-line of steel and concrete pipes buried in loose sand, dense sand and undrained clay. It must be noted that *P* is the intensity of surface pressure, *P* (*cr*, *inv* and *spr*) are the crown, invert and spring-line pressure respectively, *H* is the cover depth while *D* is the diameter of pipe, *x* is the displacement at the crown, invert and spring-line of pipes, *M* is the Young's modulus of soil, *P* is the surface pressure intensity, γ is the unit weight of soil and *r* is the radius of pipe. The various effects

considered for various blast scenarios are: For surface blast, parameters considered are the effects of intensity of dimensionless surface pressure, P/P(cr, inv and spr) against H/D ratios, intensity of dimensionless deflection, xM/PR against H/D ratios, intensity of dimensionless pressure, $P(cr, inv \text{ and } spr)/\gamma$ H against H/D ratios, intensity of dimensionless deflection, xM/γ HR against H/D ratios, dimensionless radius-to-thickness ratio, R/t against deflection, dimensionless radius-to-thickness ratio, R/t against dimensionless pressure P/γ H, soft trench material, various embedment depth ratios, H/D and liquefied soil [16, 17].

Others are: varying Young's modulus, *E* of pipe material, varying Young's modulus and *E* of soil material. For underground blast, parameters considered are the dimensionless deflection, *P(cr, inv* and *spr)/* γ *H* against *H/D* ratios, varying seismic velocity (i. e. loading wave velocity), soft trench material, various embedment depth ratios, *H/D*, liquefied soil, varying Young's modulus, *E* of pipe material, varying Young's modulus, *E* of soil material and varying seismic velocity. In the case of open trench blast, parameters considered are the effects of dimensionless pressure, *P(cr, inv* and *spr)/* γ *H* against *H/D* ratios and dimensionless deflection, *xM/* γ *HR* against *H/D* ratios. For internal explosion, parameters considered are the effects of a 50 kg TNT investigated for dimensionless pressure and *P/* γ *H* at the surface of the ground (Figure 2 a and b).



4. Results and discussion

For 250 kg TNT explosive due to surface blast, peak reflected pressure reduced from 290 x 10^3 kPa at 1 m to 18 kPa at 100 m while for the same weight of explosive, peak side-on overpressure reduced from 25.5 x 10^3 kPa at 1 m to 9 kPa at 100 m. The specific impulse due to the same weight of explosive reduced from 4.2 x 10^3 kPa-ms at 1 m to 132 kPa-ms at 100 m while shock front velocity reduced from 5 m/ms at 1 m to 0.34 m/ms at 100 m. In addition to this, for the same weight of explosive, horizontal and vertical displacement reduced from 0.5 m at 1 m to 6 x 16^{-5} m at 100 m while horizontal and vertical velocity

reduced from 16.5 m/s at 1 m to 0.16 m/s at 100 m. In addition, horizontal and vertical acceleration for sand reduced from $29.6 \times 10^3 \text{ m/s}^2$ at 1 m to 4 m/s² at 100 m while horizontal and vertical acceleration for undrained clay reduced from 79.2 x 10^3 m/s^2 to 8 m/s² for the same weight of explosive. Other weight of explosives showed similar trend. Details of these could be found in [7, 9, 11, 22] and [28].

The blast parameters in surface blast indicate that blast energy attenuates as the distance from the point of explosion increases. This is due to two reasons. Firstly, due to geometric effect and secondly due to energy dissipation as a result of work done in plastically deforming the soil matrix, i.e. absorption of energy by the soil media through inelastic deformations. The arrival time of wave energy is higher in loose and dry soils compared to saturated clay. This is due to the fact that loose sand and dry soils contains void and wave energy travels faster in air than solid medium. Saturated clay contains little or no void. In addition, clay mineral particles like montimorilonite, vermiculite, etc. are held together by Van-der-Wall force of attraction [10].

4.1. Results of blast loads for underground blast

For 250 kg TNT explosive, peak particle displacement reduced from 6.3 m at 1 m to 2.0×10^{-3} m at 100 m while peak particle velocity reduced from 606 m/s at 1 m to 2.0×10^{-3} m/s at 100 m for the same weight of explosive. In addition, for the same weight of explosive, loading wave velocity for sand and undrained clay reduced to the seismic velocity of soil at 5 m respectively. Furthermore, for the same weight of explosive, side-on overpressure for sand reduced from 3.5×10^{6} kPa at 1 m to 1.04 kPa at 100 m while the side-on overpressure for undrained clay reduced from 5.5×10^{6} kPa at 1 m to 5.6 kPa at 100 m for the same weight of explosive. Finally, for the same weight of explosive, the specific impulse for sand reduced from 36×10^{3} kPa-ms at 1 m to 1 kPa-ms at 100 m while the same weight of explosive. Other weight of explosives showed similar trend. Details of these could be found in [7, 10, 12, 26], and [33].

In the case of underground blast, soil movement parameters reduce greatly as the distance from the point of explosion increases compared to surface blast, although the impact and tremors of underground blast is felt over a wide area. This shows that ground movement parameters in surface blast has impact over a wide area compared to underground blast with tremors over a wide area and long distance. This is because for a fully buried or partially buried charge located very close to the surface of the ground, part of the energy released during underground explosion is used to plastically deform the soil, thereby forming a crater for the elasto-plastic and visco-plastic deformation of soil. The remaining energy is released into the surrounding soil which is absorbed and attenuates at a rate depending on the geotechnical properties (mostly stiffness) of the soil medium. Wave energy decreases (i. e. attenuate) as the distance increases. Attenuation can occur due to absorption of energy by the medium through inelastic deformations and by three dimensional or spatial dispersion of the air blast energy.

In underground blast, from the result of the study, the ratio of peak particle displacement for all soils to that of saturated clay is 0.135 and, the zone of influence of underground blast is relatively small but the tremors from the underground blast could be felt over a wide area. For a partially buried charge, there is blast wave, and this wave reduces to the seismic velocity of

soil as the distance increases. But for a sufficiently deep buried charge, there is no blast wave. In order to design underground pipes to resist the effects of blasts, these will help in estimating the magnitude of the blast load parameters to be used at various stand-off points up to 100 meters [22, 25, 33].

4.2. Behaviour of underground pipes due to surface blast

The results of the displacement, pressure, stress and strain at the crown, invert and spring-line of underground pipes at embedment ratios of 1 to 5 was observed. From the results, crown, invert and spring-line displacements reduce as embedment ratios increases in loose sand, dense sand and undrained clay. Displacement of between 0.0001 m and 0.00001 m observed for steel and concrete pipes respectively are zero displacements. In addition, crown, invert and spring-line pressure, stress and strain reduces at the embedment ratio of 5. In the behaviour study of the underground pipes due to blast loads, pressure changes from negative to positive in the pipes due to dilatations and compressions caused by transient stress pulse of compression wave. For steel pipe at H/D = 1, crown and invert displacement in loose sand is the highest and least in undrained clay. It is shown in this study that increasing the burial depth enhances the confinement on the underground pipe, hence reduces the observed behaviour parameters under surface blast loadings.

These results further indicate that it is necessary to evaluate the blast-resistance of underground pipes with small burial depth. It is also shown that materials undergo more behaviour and as a result yield easily and more at lower depth of burial if the yield stress is exceeded. Burial depth affected the behaviour of underground pipes under blast loading. With small burial depth of 1 m to 2 m, due to low confinement from the ground, observed parameters could be significantly large and underground structures like pipes could be severely damaged even with moderate surface blast. From the results of this study, it shows that the increase in the depth of burial increase the distance of the structure from the concentration of detonation which in turn reduce the intensity of the shockwaves hitting or striking it and will also help in reducing the structural behaviours [10, 18].

Going by the results of this study, higher observed parameters at depth of burial of 1 m and 2 m is possible because of the relatively closer distance of the structures to the centre of explosive detonation compared to the case where the pipes have been buried at 4 m to 5 m into the soil. Also, the overburden stresses as a result of the overburden pressure of soil on the pipes which act vertically downward can increase the stability of the pipes buried deeper into the soil. This reduces the responses and behaviours occurring in action to the striking blast waves. The two thicknesses (i. e. 10 mm and 20 mm) of steel and concrete pipes considered showed similar trend and behaviour in the response in terms of reduction in the observed parameters as embedment ratios increases.

4.3. Behaviours of underground pipes due to underground blast

For a given loading wave velocity, displacement in pipes due to underground blast is almost constant at 1.5 m (with a difference of 0.0005 m) at all the embedment ratios considered

irrespective of the material properties. The maximum pressure observed is 0.12 N/mm² at H/D ratio of 2 and this pressure changes from negative to positive due to reasons outlined earlier. The maximum stress observed is 0.25 N/mm² at H/D ratio of 3 and 0.30 N/mm² at H/D ratio of 2 while the maximum strain was observed to be 0.0000027 at H/D ratio of 2. The above results show that as the ground movement parameter due to underground blast, that is, peak particle velocity travels within the soil medium, it transmits the load bodily to the buried pipes along the direction of travel [13]. Reduction in pressure, stress and strain is noticeable as embedment ratios increases. Though there is reduction in all the observed parameters as the embedment ratio increases, for clarification in order to show the trend clearly, dimensional analysis was used to further analyze the results of the behaviour of underground pipes due to underground blast. In practice, if underground blast occurs, the buried pipes will fly out of the ground if the loading wave velocity is higher than the commonly used seismic velocity of soil [19].

4.4. Behaviours of underground pipes due to open trench blast

The maximum observed displacement is 0.38 mm at H/D ratio of 2. Displacement observed at the crown, invert and spring-line of underground pipes due to open trench blast is lower respectively compared to that obtained in other categories of blast. Unlike underground blast, the wave only impeaches on the side of the open trench. In addition, virtually all the parameters observed at the crown, invert and spring-line of pipes reduces at embedment ratios of 3 beyond which no significant changes occurred. The maximum observed pressure (either negative or positive as the case may be) is 0.15 N/mm² at H/D ratio of 2. This pressure also changes from negative to positive.

The maximum stress and strain observed are 0.35 N/mm² and 0.0000019 at H/D ratio of 2 respectively. The displacements, pressures, stresses and strains at the crown, invert and springline of buried pipes reduce as the embedment ratio (that is, depth of burial) increases with a sharp increase at embedment ratio of 2 [13]. All these are noticeable in loose sand, dense sand and undrained clay. The results indicate that it is more necessary to evaluate the blast-resistance of underground pipes with small depth of burial of 1 m and 2 m. The observed parameters are not as high, compared to other types of blast considered in this study [20, 23].

4.5. Behaviours of underground pipes due to internal explosion

Arrival time and duration are less important in the behaviour study of underground pipes due to internal explosion. This is because these two have to be simulated. Details of this could be found in [23]. As pipe radius increase from 0.5 m to 0.6 m, for small explosive charge, arrival time and duration of blast wave increases. But for higher explosive charge, arrival time and duration of blast wave is almost constant. As the thickness of steel and concrete pipes reduces, time history as a result of internal explosion increases in the same proportion. Depth of burial of pipes showed no significant changes in the trend of time history of external work and energies generated due to internal explosion. This is because similar trend was observed for pipes buried at 3.04 m and 6.04 m for all the time history observed.

The pressures on the ground surface in loose sand due to internal explosion in 20 mm thick steel and concrete pipes buried at 3.04 m depth are 28.78 kPa and 80.827 kPa respectively while

that of undrained clay are 7129.12 kPa and 9341.8 kPa respectively. This shows that reduction is more in loose sand due to arching effect compared to undrained clay. In terms thickness of pipes, for 20 mm thick steel pipes buried in loose sand and undrained clay at 3.04 m depth, pressure on the ground surface is 28.78 kPa and 7129.12 kPa respectively while for 10 mm thick steel pipes buried in loose sand and undrained clay at 3.04 m depth, pressure is 54.398 kPa and 13335.8 kPa respectively.

From the results of this study, it is shown that arching effect has contributed to the reduction in pressure on the ground surface while undrained clay is proven to be problematic. This study has equally shown that with increased thickness of pipe (preferably steel pipes), pressure on the ground surface due to internal explosion could be reduced. In addition, from the results of this study, steel pipes of 20 mm thick, 1.0 m diameter buried at 3.04 m depth in loose sand, dense sand and undrained clay have similar maximum values of external work to be 5.64 x 10^6 N-m at 0.000334 s, internal energy is 5.24 x 10^6 N-m at 0.000334 s, kinetic energy is 1.41 x 10^6 N-m at 0.000334 s, strain energy is 5.24×10^6 N-m at 0.000334 s and total energy is -1.81×10^5 N-m at 0.000552 s. On the other hand, concrete pipes of the same diameter and thickness buried at the same depth in different soil media as stated above have similar maximum values of external energy is 2.46×10^7 N-m at 0.000427 s, kinetic energy is 8.15×10^6 N-m at 0.000151 s, strain energy is 2.46×10^7 N-m at 0.000427 s and total energy is -2.03×10^6 N-m at 0.000646 s.

Similarly, steel pipes of 10 mm thick, 1.0 m diameter buried at 3.04 m depth in loose sand, dense sand and undrained clay have similar maximum values of external work as requested for in the model to be 1.07×10^7 N-m at 0.000332 s, internal energy is 5.02×10^7 N-m at 0.000393 s, kinetic energy is 2.70×10^6 N-m at 0.000152 s, strain energy is 9.77×10^6 N-m at 0.000332 s and total energy is -3.24×10^5 at 0.000824 s. On the other hand, concrete pipes of the same diameter and thickness buried at the same depth in different soil media stated above, have similar maximum values of external work as requested for in the model to be 5.57×10^7 N-m at 0.000378 s, internal energy is 5.03×10^7 N-m at 0.000393 s, kinetic energy is 1.66×10^7 N-m at 0.000167 s, strain energy is 5.03×10^7 N-m at 0.000393 s and total energy is -1.95×10^6 N-m at 0.001116 s. Details of these could be found in [23].

From these results, it shows clearly that, irrespective of the materials, the behavior of wave energies due to explosion in an empty underground pipe is confined explosion which is different from other types of blast scenarios. This is because the pressures is reflected from the internal surface of the empty pipe and later converge at the centre point of the explosion after which the residual wave energies travel back and impinges on the internal surface of the pipe. The geotechnical properties of soil media show no significant consequences. This justifies the non-inclusion of intervening medium in the modelling for analysis. The equivalent earthquake parameters on the surface of the ground due to 50 kg TNT explosion in underground pipes are higher than those recorded in San Fernando earthquake of 1971 [30].

Earth arching increases the load-carrying capacity of buried structures (that is, underground structures - pipes). It allows the soil to redistribute the load from the blast loads evenly into the form "arches" which eventually transfers the blast loads away from the structure. Arching effects are much greater in sands than in silts or clays. According to [35], arching is also greater in dense sands than in loose sands. If one part of the support of a soil mass yield while the remaining part stays in place, the soil adjoining the yielding part of the soil moves out of its

original position between adjacent stationary soil masses. The relative displacement within the soil mass itself is opposed by the action of the shearing resistance within the contact zone between the yielding soil masses and stationary soil masses. Since the action of the shearing resistance tends to keep or stabilize the yielding soil mass in its original position, the pressure on the yielding part of the support is reduced and the pressure on the stationary parts is increased. This process of the transfer of pressure from a yielding part of the soil mass to the adjacent non-yielding parts of a soil mass is called the arching effect [35].

4.6. Dimensionless behaviours for surface blast

Results of dimensionless intensity of pipe pressure against H/D ratio due to surface blast in different ground media are presented in Figures 3 to 8. From these results, it shows that as embedment ratio increases, pressure at the crown, invert and spring-line of underground pipes reduces more in loose sand followed by dense sand with no remarkable changes in undrained clay. This shows that arching effect has taken place in loose sand as well as dense sand. The increase in dimensionless pressure is as a result of surface pressure and overburden of soil on the pipes. From the results of dimensionless pipe deflection in terms of pipe pressure and radius of pipe due to surface blast, it is shown that as the embedment ratio increases, dimensionless pipe deflection reduces more in loose sand, but less in dense sand with no appreciable and remarkable reduction in undrained clay. In addition to this, dimensionless deflection reduced to 0.13 %, 1.9 % and 55 % at H/D ratio of 5 in loose sand, dense sand and undrained clay respectively [6, 10]. This further confirmed that undrained clay is problematic in the response of underground pipes to blast loads from surface blast, while loose sand (or loose materials like tire-chip backfilling) would mitigate the consequence of blast [16, 17].

From the results of dimensionless pipe pressure in terms of overburden of soil layer, it is shown that as the embedment ratio increases, dimensionless pipe pressure reduces more in loose sand than dense sand with little appreciable and remarkable reduction in undrained clay. From the results of dimensionless pipe deflections in terms of unit weight of soil, depth of burial of pipe as well as radius of pipe, it is shown that as embedment ratio increases, dimensionless pipe deflection reduces more in loose sand than dense sand but least in undrained clay. As pointed earlier, increasing the burial depth of underground pipes enhances and increases the confinement on underground pipes and as a result reduces the maximum stress under blast loading [13]. These results further emphasizes the indication that it is more necessary to evaluate the blast-resistance of underground pipes with relative small depth of burial of 1 m to 2 m. This is because due to blast load materials yield easily and more at lower depth of burial.

Using dimensional analysis, this study has shown that undrained clay is problematic and as a result cannot reduce the impact of surface blast loads on underground pipes (Figures 5 and 8). These clay minerals have swelling properties when in undrained condition as well as cohesive characteristics and that makes consolidation process takes longer time to complete. Mitigation measures in the form of soil stabilization and ground improvement techniques could mitigate the consequence of blast [14]. In addition to this, the results of this study have also shown that burial depth play significant role in the response of underground pipes due to surface blasts. Increasing the burial depth of underground pipe enhances the confinement on the underground pipe due to blast loads, hence reduces the maximum displacement, pressure, stress and strain under the categories of blast loading considered.

In addition, this study has also shown that loose material can reduce the impact of surface blast loads on underground pipes. Loose materials like tire-chip backfilling discussed earlier could be used to reduce the effects of surface blast loads on underground pipes. The results of dimensional for steel pipes and concrete pipes showed similar trend. There is a possibility that the stresses and strains generated due to dimensionless deflection in the underground pipes can be significantly different at different burial depths. The depth of soil cover over the buried pipes increases the over burden stresses on it (that is, overburden from the soil above the buried pipes). This can help in stabilizing it with respect to its responses and behaviours to an externally applied impulsive action such as blast loads. This can also help in the reduction of the vibrations which occur in response to an explosive blast action. Moreover, as established in this study, the increase in depth of burial increases the distance of the pipes from the centre or zone of detonation (that is, the zone of influence of the blast action) which can also reduce the intensity of the shockwaves striking it. At lower depth of burial (i. e. at H/D ratios of 1 and 2), there is relatively closer proximity of the underground pipes to the centre of explosive charge on the ground surface which is causing higher responses to a large extent in the pipes as compared to the case where the pipes have been buried deep into the soil (i. e. at H/D ratios of 4 and 5).





Figure 4. Dimensionless intensity of pipe pressure against H/D ratio for surface blast [10, 16, 17]

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Figure 5. Dimensionless intensity of pipe pressure against H/D ratio for surface blast



Figure 6. Dimensionless pipe pressure against H/D ratio for surface blast



Figure 7. Dimensionless pipe pressure against H/D ratio for surface blast



Figure 8. Dimensionless pipe pressure against H/D ratio for surface blast [10, 16, 17]

For further clarification, so that design guidelines and parameters could be established, deflection of steel pipes and concrete pipes of various thicknesses buried in undrained clay were analyzed using dimensional analysis [8]. From the results of deflection against thickness ratio of pipes buried in undrained clay, it shows that as the thickness ratio R/t increases, crown and invert deflections in concrete pipes reduces with a sharp increase at R/t ratio of 2 and 3 while spring-line deflections increase with a sharp reduction at R/t ratio of 2 and 3. In steel pipes, deflections increases as R/t ratio increases with a sharp reduction in the neighbourhood of R/t ratio of 3 and 10. With the increase of R/t ratio means decrease of thickness of pipes, hence, result in increasing the horizontal spring-line displacement, but the large increase in the thickness changes the spring-line displacement only a small amount [10, 16, 17].

4.7. Dimensionless behaviours for underground blast

Results of dimensionless intensity of pipe pressure against H/D ratio due to underground blast in different ground media are presented in Figures 9 to 11. From the result of dimensionless pipe pressure in terms of unit weight of soil and cover depth of pipe, it was observed that crown, invert and spring-line pressure reduce from embedment ratios H/D of 1 to 3 beyond which there is no significant reduction (Figure 9). The variation in the dimensionless pressure at H/D ratios of 3 to 5 is due to the dynamic nature of underground blast as well as transient stress pulse of blast wave. This clearly indicates that the minimum embedment ratio for underground steel and concrete pipes in loose sand and dense sand to reduce the impact of underground blast is 3 (i. e. 3 m) beyond which there are no remarkable or significant changes. This study has shown that increasing the burial depth of underground pipes enhances the confinement on the underground pipes, hence reduces the maximum displacement, pressure, stress and strain under blast loading. From the results of dimensionless pipe deflection in terms of unit weight of soil, cover depth of pipe as well as radius of pipe, steel and concrete pipe deflections reduce as the embedment ratio increases. This is applicable to pipes buried in loose sand and dense sand. Both steel and concrete pipes of different thicknesses show similar trend in the results. The results have shown that the dimensionless deflection reduced from 3100 at H/D ratio of 1 to 620 at H/D ratio of 5 in loose sand compared to dense sand where dimensionless deflection reduced from 8500 at *H/D* ratio of 1 to 1700 at *H/D* ratio of 5 (Figures 10 and 11). This shows that loose material can reduce moment and stress in underground pipes as a result of displacement caused by underground blast even at lower depth of burial. Typical example of loose material is tire-chip backfilling [10, 16, 17, 31, 34].



Figure 9. Dimensionless pipe pressure against H/D ratio for underground blast



Figure 10. Dimensionless pipe deflection against H/D ratio for underground blast



Figure 11. Dimensionless pipe deflection against H/D ratio for underground blast [10, 16, 17]

4.8. Dimensionless behaviours for open trench blast

Results of dimensionless intensity of pipe pressure against H/D ratio due to open trench blast in different ground media are presented in Figures 12 to 13. In the case of open trench blast, from the results of dimensionless pipe pressure and pipe deflection, it was observed for all the ground media considered that pipe dimensionless pressure and deflection increase at embedment ratio of 2 after which it reduces at embedment ratio of 3 beyond which there is no remarkable changes (Figures 12 and 13). From the above, it could be inferred that for underground pipe to resist the effects of open trench blast it should be buried at embedment ratio greater or equal to 3 (or better still, at depth of greater or equal to 3 m). For the same open trench blast, from the results of dimensionless pipe deflection, the behaviour is similar to that of dimensionless pipe pressure. If pipes are to be buried at embedment ratio of less than 2 (i. e. less than 2 m), most especially in undrained clay, the soil has to be stabilized [10].



Figure 12. Dimensionless pressure against embedment ratio for open trench blast



Figure 13. Dimensionless pipe deflection against embedment ratio for open trench blast [10, 16, 17]

4.9. Dimensionless behaviours for internal explosion in pipes

Burial depth affected significantly the maximum stress and responses in underground pipes under blast loading. With small burial depth of 1 m and 2 m, due to low confinement from ground, displacement, pressure, stress and strain could be higher up to the yielding point of underground pipes and as a result, underground pipes could be severely damaged even with modest surface blast, underground blast and open trench blast. Undrained clay is found to be problematic, hence, soil stabilization and ground improvement techniques are necessary [16, 17, 23].

4.10. Effects of relative density of sand

From the results of the dimensionless pipe deflection in loose sand for surface blast and dimensionless pipe deflection in loose sand for underground blast at the crown, invert and spring-line respectively, there is less deflection in pipes buried in loose sand compared to pipes buried in dense sand due to displacement for surface blast and underground blast respectively. It has been shown that the ratio of the dimensionless deflection of pipes (at the crown, invert and spring-line) buried in loose sand to that of dense sand at H/D ratios of 1 and 5 is 1 : 1.8 : 2.75 : 2.5 and 1 : 44 : 8 : 18 for surface blast and at H/D ratios of 1 and 5 is 1 : 2.74 for underground blast respectively. This implies that an embedded pipe is subjected to greater hazard due to blast loads when backfill is more detail and thoroughly compacted. This situation may be overcome by using softer material for backfilling. To realize a very soft backfilling, a shredded-tire trench could be employed. However, if the tire trench backfill is sufficiently large to avoid the direct interaction and contact between sandy ground and the embedded pipe, it can resist large displacement that can cause induced moment in the buried pipe [31]. Details of these could be found in [6] and [10].

5. Conclusions

In agreement with the results of dimensional analysis, reduction in the observed parameters at the crown, invert and spring-line of underground pipes is noticeable at embedment ratios of 3 to 5. Reduction in the observed parameters is more in loose sand and dense sand compared to undrained clay, and as a result of this, undrained clay is considered problematic. Loose material like tire-chip backfilling can reduce displacement caused by surface blast loads. In underground blast, for a given loading wave velocity, displacement in pipes is almost constant at all embedment ratios considered irrespective of the material properties. Irrespective of the ground media, as the seismic velocity increases, displacement increases linearly in underground blast. There are no remarkable changes in the pressure, stress and strain at the crown, invert and spring-line at low seismic velocity. Observed parameters reduced to zero at embedment ratio of 3, in open trench blast beyond which no significant changes occurred.

In internal explosion, equivalent earthquake parameters reduce more in loose sand as it approaches the ground surface. These equivalent earthquake parameters are higher compared to San Fernando earthquake of 1971 [30]. Steel pipe has stiffness to absorb the blast wave as a

result of internal explosion compared to concrete pipes. Finally, it has been shown that undrained clay is also problematic.

It is hereby recommended that mitigation measure in the form of tire-chip backfilling could be used to reduce the bending stress and moment cause by displacement due to blast. Therefore it could be provided round the buried pipes. To account for various degree and magnitude of stress and displacement, large quantities of tire-chip could be used as backfill round the buried pipe. In large clay deposit area, the soil surrounding and around the buried pipes should be grouted and any soil stabilization and ground improvement techniques could be used to improve the stiffness of the ground media. In an area prone to blast, pipes of smaller thickness should not be buried in clay deposit where there will be unpredicted heavy rainfall that may lead to saturation of clay which will eventually result to undrained condition since it takes a long period of time for consolidation in undrained clay deposit. For all the ground media studied for the various categories of blast considered, for underground pipe to resist effects of blast loads, it should be buried at embedment ratio of not less than 3 (i. e. 3 m). However, if pipe is to be buried at embedment ratio of less than 3 (i. e. 3 m), then it is more necessary to evaluate the blast-resistance of such underground pipe with small burial depth. If pipes are to be buried directly underneath a multi-storey building or road pavement and it is resist internal explosion, for all the ground media considered, steel pipes of not less than 20 mm thick should be used.

Finally for further study, it is hereby recommended that studies should be conducted on the response of underground structures (i. e. pipes carrying fluid) due to blast, especially blast that take place on the ground surface in the case of surface blast. In addition, in the case of underground blast, research efforts should be geared towards blast that take place within the vicinity of the underground structures (i. e. pipes). In that case, non-linear, inelastic, non-homogeneous and anisotropic materials should be considered. The limitation of ABAQUS numerical code is that it cannot be used for the analysis of problems involving fluid-pipe interaction. Other numerical tools that would incorporate fluid-pipe interaction apart from ABAQUS could be used for the study.

In line with the objectives of this study, blast load parameters for various blast scenarios considered have been determined. In addition to this, the dynamic and dimensionless responses between the various components of blast have been determined. Furthermore, parametric studies were carried out and finally the establishment of design parameters and guidelines for the design of underground pipes to resist the effects of blast loads [36]. Consequently environmental risks and hazards caused by blast would be greatly reduced if not completely eliminated.

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Author details

Akinola Johnson Olarewaju

Address all correspondence to: akinolajolarewaju@yahoo.com

The Federal Polytechnic Ilaro, Ogun State, Nigeria

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